



SOIL MECHANICS



REVIEW OF SOIL MECHANICS, SOIL BEHAVIOR, & GEOTECHNICAL SITE INVESTIGATIONS SECTION 2

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SYMBOLS USED IN THIS SECTION

SOIL MECHANICS

| | | |
|------------------|---|------|
| w_n | Moisture Content | 2-6 |
| M | Mass | 2-5 |
| V | Volume | 2-6 |
| S | Degree of Saturation | 2-6 |
| e | Void Ratio | 2-6 |
| n | Porosity | 2-6 |
| γ_d | Dry Unit Weight (Dry Density) | 2-6 |
| γ_t | Wet (Total) Unit Weight (Wet Density) | 2-6 |
| γ_s | Saturated Unit Weight (Saturated Density) | 2-6 |
| γ' | Submerged Unit Weight (Submerged Density) | 2-6 |
| S_t | Soil Sensitivity | 2-8 |
| SL | Shrinkage Limit | 2-7 |
| PL | Plastic Limit | 2-7 |
| LL | Liquid Limit | 2-7 |
| PI | Plasticity Index | 2-7 |
| $L.I.$ | Liquidity Index | 2-7 |
| K_t | Torque Multiplier for Helical Piles/Anchors | 2-8 |
| σ' | Effective Stress | 2-11 |
| σ | Total Stress | 2-11 |
| u | Pore Water Pressure | 2-11 |
| τ_f | Shear Strength | 2-12 |
| ϕ | Friction Angle | 2-12 |
| c | Cohesion | 2-11 |
| s_u | Undrained Shear Strength | 2-11 |

| | | |
|----------------------|--|------|
| CFA | Continuous Flight Auger | 2-15 |
| HSA..... | Hollow Stem Auger | 2-15 |
| GWT | Ground Water Table | 2-13 |
| SPT | Standard Penetration Test | 2-16 |
| SS..... | Split Spoon | 2-16 |
| ST..... | Shelby Tube | 2-16 |
| CPT | Cone Penetration Test | 2-18 |
| CPTU..... | Piezocene Penetration Test | 2-18 |
| q _u | Unconfined Compressive Strength | 2-22 |
| UC..... | Unconfined Compression Test | 2-22 |
| OCR..... | Overconsolidation Ratio | 2-17 |
| N | Field Blowcount Value from Standard Penetration Test | 2-17 |
| DMT | Dilatometer Test | 2-18 |
| FVT | Field Vane Test | 2-19 |
| RQD | Rock Quality Designation | 2-20 |
| USCS..... | Unified Soil Classification System | 2-6 |
| VST | Vane Shear Test | 2-19 |

DISCLAIMER

The information in this manual is provided as a guide to assist you with your design and in writing your own specifications.

Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis and consulting state and local building codes and authorities should be conducted prior to any installation to ascertain and verify compliance to relevant rules, regulations and requirements.

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INTRODUCTION

The use of manufactured steel foundation products generally requires a prior geotechnical investigation of the subsurface condition of the foundation soils at the site of a proposed project. In addition to the geotechnical investigation, it is necessary to define the structural load requirements and required Factor of Safety (FS) for use in the overall design approach. CHANCE® Civil Construction manufactures or supplies two main lines of steel foundation products:

- ATLAS RESISTANCE® piers for underpinning and repair of residential and commercial buildings, retaining structures and slabs.
- CHANCE® Helical Piles for new construction and repair of residential and commercial buildings; CHANCE® Helical Tiebacks and a SOIL SCREW® Retention System used in excavation shoring systems, retaining walls and slope stabilization; and CHANCE® Helical Anchors are utilized for communication towers, transmission & distribution power lines, signs, light standards and commercial buildings subject to wind and earthquake load.

SOIL MECHANICS

Terzaghi stated in his book Theoretical Soil Mechanics (1943): “. . . the theories of soil mechanics provide us only with a working hypothesis, because our knowledge of the average physical properties of the subsoil and of the orientation of the boundaries between the individual strata is always incomplete and often utterly inadequate. Nevertheless, from a practical point of view, the working hypothesis furnished by soil mechanics is as useful as the theory of structures in other branches of civil engineering.”

Advance planning and careful observation by the engineer during the construction process can help fill the gaps between working hypothesis and fact. The intent of this section of the Design Manual is to provide a basic background or review of soil mechanics so the engineer can develop a useful “working hypothesis” for the design and use of CHANCE® Helical Piles and ATLAS RESISTANCE® Piers.

THE SOIL PROFILE

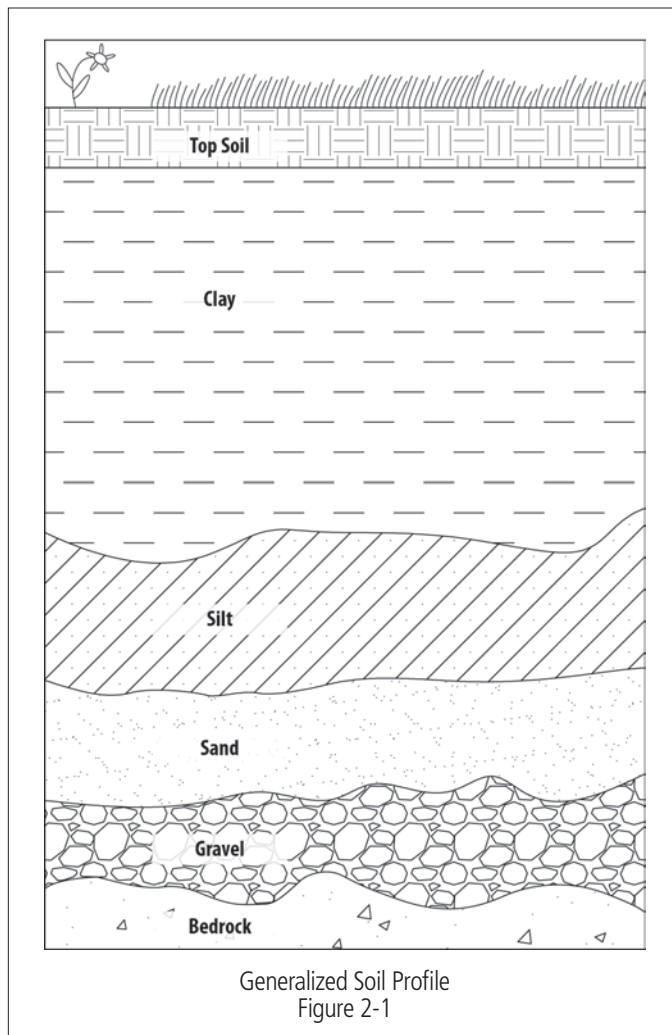
Rock or soil material, derived by geologic processes, are subject to physical and chemical changes brought about by the climate and other factors prevalent at the location of the rock or soil material. Vegetation, rainfall, freeze/thaw cycles, drought, erosion, leaching, and other natural processes result in gradual but profound changes in the character of the soil over the passage of time. These processes bring about the soil profile.

The soil profile is a natural succession of zones or strata below the ground surface. It may extend to various depths, and each stratum may have various thicknesses. The upper layer of the profile is typically rich in organic plant and animal residues mixed with a given mineral-based soil. Soil layers below the topsoil can usually be distinguished by a contrast in color and degree of weathering. The physical properties of each layer usually differ from each other. Topsoil is seldom used for construction. Figure 2-1 shows a typical generalized soil profile.

Deeper layers will have varying suitability depending on their properties and location. It is important to relate engineering properties to individual soil layers in order for the data to be meaningful. If data from several layers of varying strength are averaged, the result can be misleading and meaningless. Equally misleading is the practice of factoring a given soil’s engineering properties for design. This can lead to overly conservative foundation design.

DEFINITION of SOIL

Soil is defined as sediments or other accumulation of mineral particles produced by the physical or chemical disintegration of rock, plus the air, water, organic matter, and other substances that may be included. Soil is typically a non-homogeneous, porous, earthen material whose engineering behavior is influenced by changes in composition, moisture content, degree of saturation, density, and stress history.



The origin of soil can be broken down to two basic types: residual and transported. Residual soil is produced by the in-place weathering (decomposition) of rock by chemical or physical action. Residual soils may be very thick in areas of intense weathering such as the tropics, or they may be thin or absent in areas of rapid erosion such as steep slopes. Residual soils are usually clayey or silty, and their properties are related to climate and other factors prevalent at the location of the soil. Residual soils are usually preferred to support foundations, as they tend to have better and more predictable engineering properties.

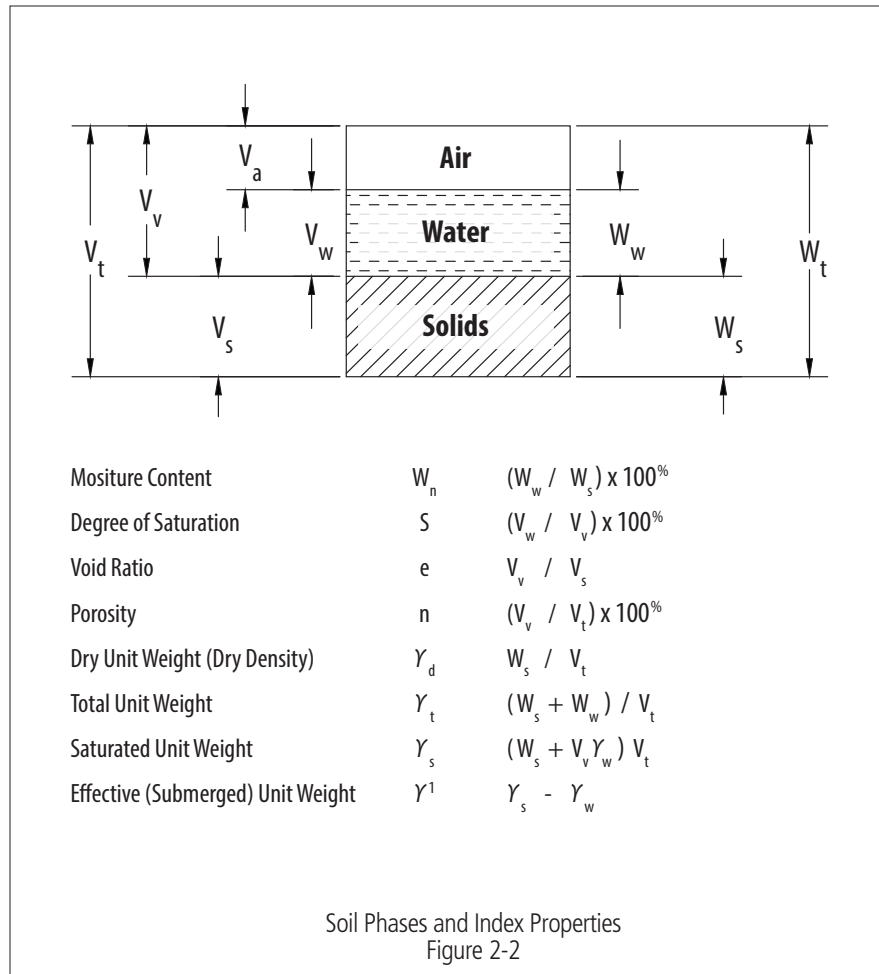
Transported or deposited soils are derived by the movement of soil from one location to another location by natural means. The means are generally wind, water, ice, and gravity. The character of the resulting deposit often reflects the modes of transportation and deposition and the source material. Deposits by water include alluvial floodplains, coastal plains, and beaches. Deposits by wind include sand dunes and loess. Deposits by melting ice include glacial till and outwash. Each of these materials has behavioral characteristics dependent on geological origin, and the geological name, such as loess, conveys much useful information. Transported soils – particularly by wind or water – can be of poor quality in terms of engineering properties.

A soil mass is a porous material containing solid particles interspersed with pores or voids. These voids may be filled with air, water, or both. Figure 2-2 shows a conceptual block diagram of relative volumes of air, water, and soil solids in a given volume of soil. Pertinent volumes are indicated by symbols to the left while weights of these material volumes are indicated by

symbols to the right. Figure 2-2 also provides several terms used to define the relative amounts of soil, air, and water in a soil mass. Density is the mass of a unit volume of soil. It is more correctly termed the unit weight. Density may be expressed either as a wet density (including both soil and water) or as a dry density (soil only). Moisture content is the ratio of the weight of water to the weight of soil solids expressed as a percent. Porosity is the ratio of the volume of voids to the total volume of the soil mass regardless of the amount of air or water contained in the voids. Void ratio is the ratio of the volume of voids to the volume of solids.

The porosity and void ratio of a soil depend upon the degree of compaction or consolidation. For a particular soil in different conditions, the porosity and void ratio will vary and can be used to judge relative stability and load-carrying capacity – i.e., stability and load capacity increase as porosity and void ratio decrease. If water fills all the voids in a soil mass, the soil is said to be saturated, i.e., $S = 100\%$.

Permeability or hydraulic conductivity is the property of soil that allows it to transmit water. Its value depends largely on the size and number of the void spaces, which in turn depends on the size, shape, and state of packing of the soil grains. A clay soil can have the same void ratio and unit weight as a sand soil, but the clay will have a lower permeability because of the much smaller pores or flow channels in the soil structure. Water drains slowly from fine-grained soils like clays. As the pore water drains, clays creep, or consolidate slowly over time. Sands have high permeability, thus pore water will drain quickly. As a result, sands will creep, or consolidate quickly when loaded until the water drains. After drainage, the creep reduces significantly.



BASIC SOIL TYPES

As stated above, soil is typically a non-homogeneous material. The solid mineral particles in soils vary widely in size, shape, mineralogical composition, and surface-chemical characteristics. This solid portion of the soil mass is often referred to as the soil skeleton, and the pattern of arrangement of the individual particles is called the soil structure.

The sizes of soil particles and the distribution of sizes throughout the soil mass are important factors which influence soil properties and performance. There are two basic soil types that are defined by particle size. The first type is coarse-grained soils. Coarse-grained soils are defined as soil that have 50% or more particles retained by the #200 sieve (0.074 mm). The #200 sieve has 200 openings per inch.

Coarse-grained soils consist of cobbles, gravels, and sands. Coarse-grained soils are sometimes referred to as granular or cohesionless soils. The particles of cohesionless soils typically do not stick together

except in the presence of moisture, whose surface tension tends to hold particles together. This is commonly referred to as apparent cohesion.

The second type of soil is fine-grained soil. Fine-Grained soils consist of soils in which 50% or more of the particles are small enough to pass through the #200 sieve. Typical Fine-Grained soils are silts and clays. Silt particles typically range from 0.074 to 0.002 mm. Clay particles are less than 0.002 mm. It is not uncommon for clay particles to be less than 0.001 mm (colloidal size). Fine-grained soils are sometimes referred to as cohesive soils. The particles of cohesive soils tend to stick together due to molecular attraction.

For convenience in expressing the size characteristics of the various soil fractions, a number of particle-size classifications have been proposed by different agencies. Table 2-1 shows the category of various soil particles as proposed by the Unified Soil Classification System (USCS), which has gained wide recognition.

An effective way to present particle size data is to use grain-size distribution curves such as shown in Figure 2-3. Such curves are drawn on a semi-logarithmic scale, with the percentages finer than the grain size shown as the ordinate on the arithmetic scale. The shape of such curves shows at a glance the general grading characteristics of soil. For example, the dark line on Figure 2-3 represents a "Well-Graded" soil – with particles in a wide range. Well-graded soils consist of particles that fall into a broad range of sizes class, i.e., gravel, sand, silt-size, clay-size, and colloidal-size.

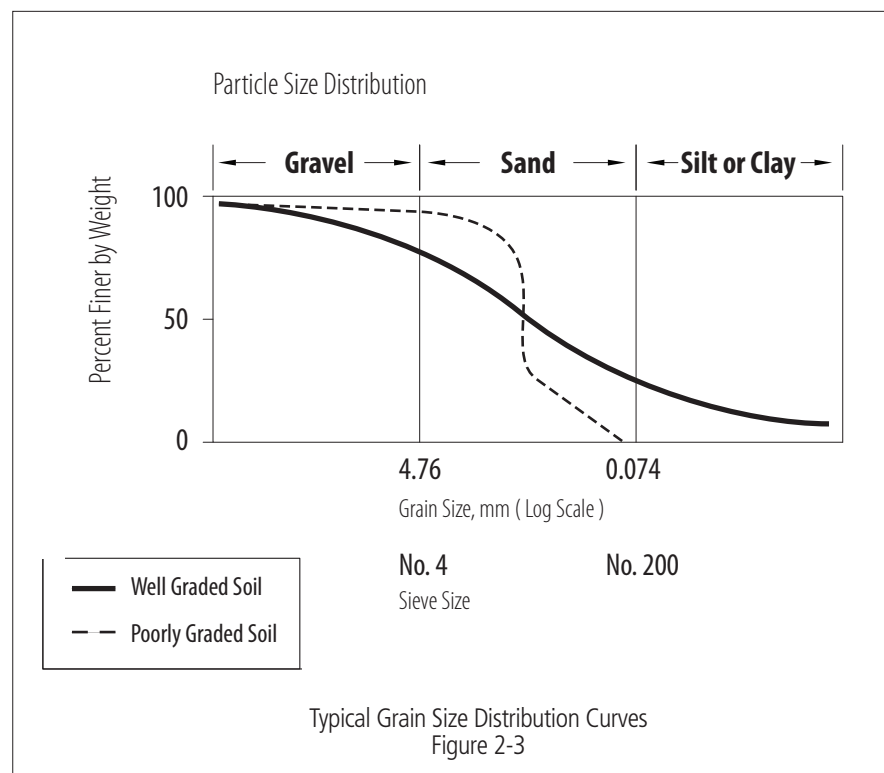
Soil Particle Sizes, Table 2-1

| PARTICLE SIZE TERM | FRACTION | SIEVE SIZE | DIAMETER | FAMILIAR REFERENCE |
|-------------------------|--------------------------|---|---|------------------------------------|
| Boulders | --- | 12" Plus | 300 mm Plus | Volleyball |
| Cobbles | --- | 3"-12" | 75 - 300 mm | Baseball |
| Gravels | Coarse Fine | 0.75" - 3" No. 4 - 0.75" | 19 - 75 mm 4.76 - 19 mm | Marbles & Peas |
| Sand | Coarse Medium Fine | No. 10 - No. 4 No. 40 - No. 10 No. 200 - No. 40 | 2 - 4.76 mm 0.42 - 2 mm 0.074 - .042 mm | Rock Salt, Table Salt, Sugar |
| Fines (silts and clays) | --- | Passing No. 200 | 0.074 mm | Flour |

SOIL CONSISTENCY STATES and INDEX PROPERTIES

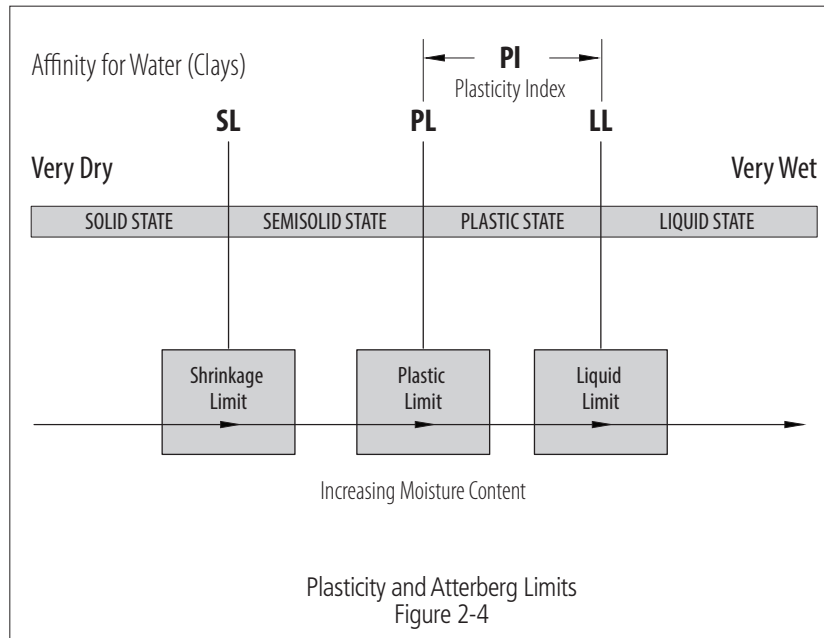
The consistency of fine-grained soils can range from a dry solid condition to a liquid form with successive addition of water and mixing as necessary to expand pore space for acceptance of water. The consistency passes from solid to semi-solid to plastic solid to viscous liquid as shown in Figure 2-4. In 1911, Atterberg, a Swedish soil scientist, defined moisture contents representing limits dividing the various states of consistency. These limits are known as Atterberg Limits. The shrinkage limit (SL) separates solid from semisolid behavior, the plastic limit (PL) separates semisolid from plastic behavior, and the liquid limit (LL) separates plastic from liquid state. Soils with water content above the liquid limit behave as a viscous liquid.

The width of the plastic state (LL-PL), in terms of moisture content, is defined as the plasticity index (PI). The PI is an important indicator of the plastic behavior a soil will exhibit. The Casagrande Plasticity Chart, shown in Figure 2-5, is a good indicator of the differences in plasticity that different fine-grained soils can have. The softness of saturated clay can be expressed numerically by the liquidity index (L.I.) defined as $L.I. = (w_n - P.L.)/(L.L. - P.L.)$. Liquidity Index is



a very useful parameter to evaluate the state of natural fine-grained soils and only requires measurement of the natural water content, the Liquid Limit and the Plastic Limit. Atterberg limits can be used as an approximate indicator of stress history of a given soil. Values of L.I. greater than or equal to one are indicative of very soft sensitive soils. In other words, the soil structure may be converted into a viscous fluid when disturbed or remolded by pile driving, caisson drilling, or the installation of CHANCE® Helical Piles/Anchors, or ATLAS RESISTANCE® Piers.

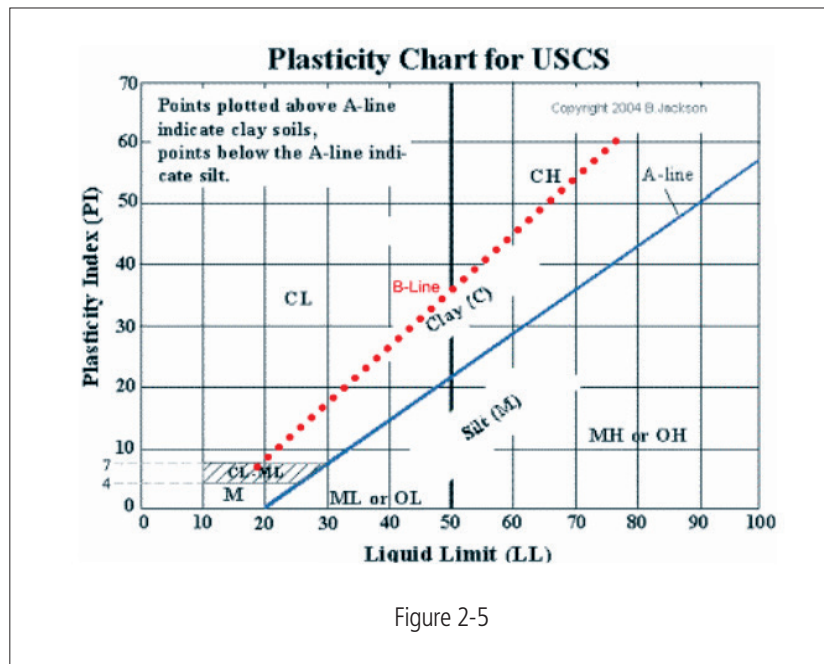
If the moisture content (w_n) of saturated clay is approximately the same as the L.L. ($L.I. = 1.0$), the soil is probably near normally consolidated. This typically results in an empirical torque multiplier for helical piles/anchors (K_t) = 10. If the w_n of saturated clay is greater than the L.L.



(L.I. > 1.0), the soil is on the verge of being a viscous liquid and K_t will be less than 10. If the w_n of saturated clay is close to the P.L. (L.I. = 0), the soil is dry and overconsolidated and K_t typically ranges between 12 and 14. If the w_n of a saturated clay is intermediate (between the PL and LL), the soil is probably over consolidated and K_t will be above 10. Many natural fine-grained soils are over consolidated, or have a history of having been loaded to a pressure higher than exists today. Some common causes are desiccation, the removal of overburden through geological erosion, or melting of overriding glacial ice.

Clays lying at shallow depth and above the water table often exhibit overconsolidated behavior known as desiccation. They behave as overconsolidated, but the overburden pressure required has never existed in the soil. Desiccated clays are caused by an equivalent internal tension resulting from moisture evaporation. This is sometimes referred to as negative pore pressure. The problems with desiccated or partly dry expansive clay are predicting the amount of potential expansion and the expansion or swell pressure so that preventive measures can be taken.

Sensitivity of fine grained soils is defined as the ratio of the undrained shear strength of a saturated soil in the undisturbed state to that of the soil in the remolded state $S_t = s_{u_{und}}/s_{u_{rem}}$. Most clays are sensitive to some degree, but highly sensitive soils cannot be counted on for shear strength after a CHANCE® Helical Pile, ATLAS RESISTANCE® Pier, drilled shaft, driven pile, etc. has passed through it. Some soils are "insensitive", that is, the remolded strength is about the same as the undisturbed strength. Highly sensitive



soils include marine deposited in a salt water environment and subsequently subjected to flushing by fresh water. Typical values of soil sensitivity are shown in Table 2-2.

ENGINEERING SOIL CLASSIFICATION

The engineering soil classification commonly used by Geotechnical Engineers is the Unified Soil Classification System (USCS). The Unified System incorporates the textural characteristics of the soil into engineering classification and utilizes results of laboratory grain-size data and Atterberg Limits shown in Table 2-1. The basics of the system are shown in Table 2-4. All soils are classified into 15 groups, each group being designated by two letters. These letters are abbreviations of certain soil characteristics as shown in Table 2-3.

Sensitivity of Soils, Table 2-2

| Soil TYPE | Description | Sensitivity |
|---|--------------------|-------------|
| Overconsolidated, Low to Medium Plastic Clays & Silty Clays | Insensitive | 1-3 |
| Normally Consolidated, Medium Plastic Clays | Medium Sensitivity | 4-8 |
| Marine Clays | Highly Sensitive | 10-80 |

USCS Soil Group Symbol Characteristics, Table 2-3

| | 1st Symbol | | 2nd Symbol |
|----|-------------------------------------|---|-------------------|
| G | Gravel | O | Organic |
| S | Sand | W | Well Graded |
| M | Non-plastic or Low Plasticity Fines | P | Poorly Graded |
| C | Plastic Fines | L | Low Liquid Limit |
| Pt | Peat, Humus, Swamp Soils | H | High Liquid Limit |

COARSE-GRAINED SOILS (G & S)

GW and SW groups comprise well-graded gravelly and sandy soils that contain less than 5% of non-plastic fines passing the #200 sieve. GP and SP groups comprise poorly graded gravels and sands containing less than 5% of non-plastic fines. GM and SM groups generally include gravels or sands that contain more than 12% of fines having little or no plasticity. GC and SC groups comprise gravelly or sandy soils with more than 12% of fines, which exhibit either low or high plasticity.

FINE-GRAINED SOILS (M & C)

ML and MH groups include the predominately silty materials and micaceous or diatomaceous soils. An arbitrary division between the two groups is where the liquid limit is 50. CL and CH groups comprise clays with low (L.L. < 50) and high (L.L. > 50) liquid limits, respectively. They are primarily inorganic clays. Low plasticity clays are classified as CL and are usually lean clays, sandy clays, or silty clays. Medium-plasticity and high plasticity clays are classified as CH.

ORGANIC SOILS (O & Pt)

OL and OH groups are characterized by the presence of organic matter, including organic silts and clays. The Pt group is highly organic soils that are very compressible and have undesirable construction characteristics. Peat, humus, and swamp soils with a highly organic texture are typical.

Classification of a soil in the United Soil Classification System will require laboratory tests to determine the critical properties, but a tentative field classification is often made by drillers, geologists, or engineers; but considerable skill and experience are required. Soil boring logs often include the engineering classification of soils as described by the USCS.

Specifics of the Unified Soil Classification System (USCS), Table 2-4

| Major Divisions | | | Group Symbols | Typical Descriptions |
|--|--|---------------------|--|---|
| Coarse Grained Soils- more than 50% retained on #200 sieve.* | Gravels - 50% or more of coarse fraction retained on #4 sieve. | Clean Gravels | GW | Well-graded gravels and gravel-sand mixtures. Little or no fines. |
| | | | GP | Poorly graded gravels and gravel-sand mixtures. Little or no fines. |
| | | Gravels with Fines. | GM | Silty gravels. Gravel-sand-silt mixtures. |
| | | | GC | Clayey gravels. Gravel-sand-clay mixtures. |
| | Sands - 50% or more of coarse fraction passes #4 sieve. | Clean Sands. | SW | Well-graded sands and gravelly sands. Little or no fines. |
| | | | SP | Poorly graded sands and gravelly sands. Little or no fines. |
| | | Sand with Fines | SM | Silty sands. Sand-silt mixtures. |
| | | | SC | Clayey sands. Sand-clay mixtures. |
| Fine-Grained Soils - 50% or more passes #200 sieve.* | Silts and Clays - Liquid limit less than 50. | ML | Inorganic silts, very fine sands, rock flour, silty or clayey find sands. | |
| | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. | |
| | | OL | Organic silts and organic silty clays of low plasticity. | |
| | Silts and Clays - Liquid limit 50 or more | MH | Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts. | |
| | | CH | Inorganic clays of high plasticity, fat clays. | |
| | | OH | Organic clays of medium to high plasticity. | |
| Highly Organic Soils. | | | PT | Peat, muck and other highly organic soils. |
| *Based on the material passing the 3" (76 mm) sieve. | | | | |

*Based on the material passing the 3" (76 mm) sieve.

EFFECTIVE STRESS and PORE WATER PRESSURE

The total stress within a mass of soil at any point below a water table is equal to the sum of two components, which are known as effective stress and pore water pressure. Effective stress is defined as the total force on a cross section of a soil mass which is transmitted from grain to grain of the soil, divided by the area of the cross section, including both solid particles and void spaces. It sometimes is referred to as inter-granular stress. Pore water pressure is defined as the unit stress carried by the water in the soil pores in a cross section. Effective stress governs soil behavior and can be expressed as:

$$\sigma' = \sigma - u$$

Equation 2-1

where: σ' = the effective stress in the soil

σ = total (or applied) stress

u = pore water pressure

SOIL STRENGTH

One of the most important engineering properties of soil is its shearing strength, or its ability to resist sliding along internal surfaces within a given mass. Shear strength is the property that materially influences the bearing capacity of a foundation soil and the design of CHANCE® Helical Piles/Anchors, or ATLAS RESISTANCE® Piers. The basic principle is similar in many respects to an object that resists sliding when resting on a table.

The shear strength is the maximum shear resistance that the materials are capable of developing. Shear strength of soil consists of two parts. The first part is the friction between particles (physical property). The second part is called cohesion, or no-load shear strength due to a chemical bond between particles.

DRAINED SHEAR STRENGTH

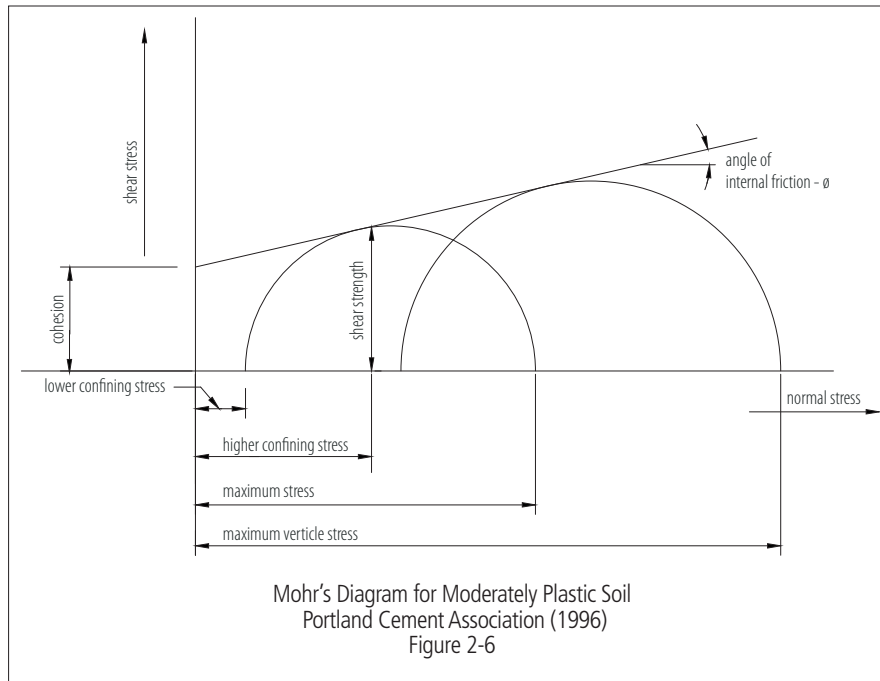
Most unsaturated coarse-grained soils and some mixed grain soils, have sufficiently high permeability that applied loads do not generate pore water pressures or any pore water pressures can dissipate during shear. This is also true if the load is applied very slowly and water is allowed to drain. The shear strength of these soils generally consists of both a "cohesive" component and a "frictional" component so that the shear strength may be reasonably described by the Mohr-Coulomb equation as shown in Equation 2-3.

UNDRAINED SHEAR STRENGTH

Saturated fine-grained soils, such as clays and silty clays subjected to rapid loading have a low enough permeability that excess pore water pressures cannot dissipate during shear. The behavior of these soils is controlled by undrained shear strength. The strength is composed of only a "cohesive" component and not a "frictional" component. The strength of these soils, is sometimes called "cohesion" (c), but a better term is simply undrained shear strength, s_u . The undrained shear strength is controlled by stress history, stress path, loading rate and vertical effective stress.

ANGLE of INTERNAL FRICTION

The shear strength of coarse-grained soils, such as sands, gravels and some silts, is closely analogous to the frictional resistance of solids in contact. The relationship between the normal stress acting on a plane in the soil and its shearing strength can be expressed by the following equation, in terms of stress:



$$\tau = \sigma \tan \phi$$

Equation 2-2

where: τ = the shearing stress at failure, or the shear strength

σ = normal stress acting on the failure plane

ϕ = friction angle

The internal friction of a given soil mass is related to the sliding friction between individual soil grains and the interlocking of soil particles. Shear strength attributable to friction requires a normal force (σ), and the soil material must exhibit friction characteristics, such as multiple contact areas. In dense soils, the individual soil grains can interlock, much like the teeth of two highly irregular gears. For sliding to occur, the individual grains must be lifted

over one another against the normal stress (σ). Therefore, the force required to overcome particle interlock is proportional to the normal stress, just the same as sliding friction is proportional to normal stress. In soil mechanics, ϕ is designated the angle of internal friction, because it represents the sum of sliding friction plus interlocking. The angle of internal friction (ϕ) is a function of density, roundness or angularity, and particle size.

COHESION

When saturated clay is consolidated, that is, when the volume of voids decreases as a result of water being squeezed out of the pores, the shear strength increases with normal stress. If the shear strength of clays which have a previous history of consolidation (i.e., pre-consolidated) is measured, the relationship between shear strength and normal stress is no longer a line intersecting the ordinate at zero. The clays exhibit a memory, or cohesive shear strength. In other words, the clays remember the pre-consolidation pressure they were previously subjected to. This means considerable shear strength is retained by the soil. Figure 2-6 is an example of the relationship between shear strength and normal stress for a pre-consolidated plastic clay as derived from a triaxial shear test. The intersection of the line at the ordinate is called the cohesion.

Cohesion is analogous to two sheets of flypaper with their sticky sides in contact. Considerable force is required to slide one over the other, even though no normal stress is applied. Cohesion is the molecular bonding or attraction between soil particles. It is a function of clay mineralogy, moisture content, particle orientation (soil structure), and density. Cohesion is associated with fine grain materials such as clays and some silts.

COULOMB EQUATION for SHEAR STRENGTH

The equation for shear strength as a linear function of total stress is called the Coulomb equation because it was first proposed by Coulomb in 1773.

$$\tau_f = c + \sigma \tan \phi$$

Equation 2-3

In terms of effective stress:

$$\tau_f = c' + (\sigma - u) \tan \phi'$$

Equation 2-4

where: τ_f = shear strength at failure
 c' = cohesion
 σ = total stress acting on the failure plane
 ϕ' = friction angle
 u = pore water pressure

Equations 2-3 and 2-4 are two of the most widely used equations in geotechnical engineering, since they approximately describe the shear strength of any soil under drained conditions. They are the basis for bearing capacity Equations 5-6 and 5-31 presented in Section 5.

SITE INVESTIGATIONS

To this point, various definitions, identification properties, limit states, engineering classifications, and soil strength properties have been discussed. This section details some of the more common soil exploration methods used to determine these various soil parameters.

The primary purpose of a geotechnical site investigation is to identify the subsurface stratification, and the key soil properties for design of the steel foundation elements. Such studies are useful for the following reasons:

ATLAS RESISTANCE® Piers:

- To locate the depth of a suitable bearing stratum for end bearing support of the underpinning pier.
- To establish the location of any weak or potentially liquefiable soil zones in which column stability of the pier shaft must be considered.
- To determine if there are any barriers to installing the pier to the required depth such as rubble fill, boulders, zones of chert or other similar rock, voids or cavities within the soil mass, any of which might require pre-drilling.
- To do a preliminary evaluation of the corrosion potential of the foundation soils as related to the performance life of the steel pier.

CHANCE® Helical Piles/Anchors, Tiebacks and SOIL SCREW® Anchors:

- To locate the depth and thickness of the soil stratum suitable for seating the helical plates of the pile and to determine the necessary soil strength parameters of that stratum.
- To establish the location of weak zones, such as peat type soils, or potentially liquefiable soils in which column stability of the pile for compression loading situations may require investigation.
- To locate the depth of the groundwater table (GWT).
- To determine if there are any barriers to installing the piles to the required depth such as fill, boulders or zones of cemented soils, or other conditions, which might require pre-drilling.
- To do a preliminary evaluation of the corrosion potential of the foundation soils as related to the performance life of the steel pile.

The extent to which a soil exploration program should reach depends on the magnitude of the project. If the proposed construction program involves only a small expenditure, the designer cannot afford to include more in the investigation than a small number of exploratory borings, test pits or helical trial probe piles and a few classification tests on representative soil samples. The lack of information about subsoil conditions must be compensated for by using a liberal factor of safety. However, if a large-scale construction operation is to be carried out under similar soil conditions,

the cost of a thorough and elaborate subsoil investigation is usually small compared to the savings that can be realized by utilizing the results in design and construction, or compared to the expense that would arise from a failure due to erroneous design assumptions. The designer must be familiar with the tools and processes available for exploring the soil, and with the methods for analyzing the results of laboratory and field tests.

A geotechnical site investigation generally consists of four phases: (1) Reconnaissance and Planning, (2) Test Boring and Sampling Program, (3) Laboratory Testing, and (4) a Geotechnical Report. A brief description of the requirements and procedures, along with the required soil parameters used in designing manufactured steel foundation products, is given in the following sections.

INITIAL RECONNAISSANCE and PLANNING

The first step in any subsoil exploration program should be an investigation of the general geological character of the site. The more clearly the site geology is understood, the more efficiently the soil exploration can be performed.

Reconnaissance and Planning includes: (1) review of the proposed project and structural load requirements and size of the structure and whether the project is new construction or structure repair, (2) a review of the general soil and geologic conditions in the proximity of the site, and (3) a site visit to observe topography and drainage conditions, rock outcrops if present, placement of borings, evidence of soil fill, including rubble and debris and evidence of landslide conditions. The planning portion includes making a preliminary determination of the number and depth of each boring as well as determining the frequency of soil sampling for laboratory testing and requesting the marking of all utilities in the zone in which borings will be conducted. Indicated below are recommended guidelines for determining the number of borings and the depth to which the boring should be taken based on the project type.

Minimum Number of Test Boring(s)

Whether the project involves underpinning/repair of an existing structure or new construction, borings should be made at each site where helical piles or resistance piers are to be installed. The recommended minimum number of borings necessary to establish a foundation soil profile is given below:

- Residential Home - One (1) boring for every 100 to 150 lineal feet of foundation.
- Commercial Building - One (1) boring for every 50 to 100 lineal feet for multistory-story structures, and every 100 to 150 lineal feet of foundation for other commercial buildings, warehouses and manufacturing buildings.
- Communication Towers - One (1) boring for each location of a cluster of piles or anchors, and one (1) boring at the tower center foundation footing.
- Sheet Pile/Earth Stabilization for Earth Cuts - One (1) boring for every 200 to 400 feet of project length.
- If the project is small or when the project has a restricted budget, helical trial probe piles installed at the site can provide information regarding the depth to the bearing strata and pile capacity.
- Or, boring number can be based on the overall project area, or based on minimum requirements per applicable building codes.

Depth of Test Boring(s)

The depth of each boring will vary depending on the project type, magnitude of foundation loads and area extent of the project structure. Some general guidelines for use in estimating required boring depths are given below:

- Residential Home - At least 15 feet deep with final 5 feet into good bearing stratum, generally "N" > 8 to 10 (See next section "Test Boring and Sampling Program" for a description of Standard Penetration Test and "N" values.)
- Commercial Building - For a single story structure at least 20 feet deep with final 5 to 10 feet into good bearing stratum (generally "N" > 15); add 5 foot depth for each additional story.



Auger Drilling Operation
Figure 2-7

- Communication Towers - Minimum of 35 feet for towers over 100 feet tall and at least 20 feet into a suitable bearing stratum (typically medium dense to dense for sands and stiff to very stiff for clays) for helical anchors/piles. The suitable bearing stratum should have a minimum “N” value of 12 for sands and a minimum of 10 for cohesive soils.
- Sheet Piling/Earth Stabilization - Boring should be taken to a depth that is at least as deep as the structure (sheet pile, retaining wall, etc.) to be anchored or until a suitable stratum is reached for seating the helical plates of the tiebacks (generally medium or denser sand or stiff clays).
- Active Seismic Areas - Depth per local codes.

TEST BORING and SAMPLING PROGRAM

In some cases, especially for small projects and shallow conditions, test borings may be conducted using hand augers or other portable equipment. In most cases, however, the site investigation will typically require drilling using a truck mounted drill rig.

The second step of the site investigation is to make exploratory boreholes or test pits that furnish more specific information regarding the general character and thickness of the individual soil strata. This step and an investigation of the general geological character of the site are recommended minimums. Other steps depend on the size of the project and the character of the soil profile.

Method of Boring and Frequency of Sampling

Drilling is typically the most economical and most expedient procedure for making borings although test pits can be an alternative for some projects. Three common types of borings obtained using truck or track mounted drill rigs are 1) wash borings (mud rotary), and 2) solid-stem continuous flight (CFA) auger drilling and 3) hollow stem flight auger (HSA) drilling. Any one of the three can be used, but CFA auger drilling is the most common – particularly for shallow borings. Wash borings or mud rotary drilling use casings to hold the borehole open and a drilling fluid to bring solid cuttings to the surface. The casing is either driven with a hammer or rotated mechanically while the hole is being advanced. The cutting bit and drill rods are inserted inside the casing and are rotated manually or mechanically. The cuttings allow the driller to visually classify the soil as to its type and condition and record the data on a log sheet at the depth of the cutting bit. Wash borings typically use water or drilling mud such as bentonite slurry depending on the soil. In some soil profiles, drilling mud prevents caving, making full-length casing unnecessary. While drilling proceeds, the driller observes the color and appearance of the mixture of soil and water/mud. This enables the driller to establish the vertical sequence of the soil profile. At 5 ft (1.5 m) intervals, or when a change in strata is noticed, the cutting bit is removed and a spoon sample is taken.

Auger drilling typically uses a continuous solid-stem flight auger rotated mechanically while the hole is being advanced. The continuous flight auger (CFA) often includes a hollow stem, which acts as a casing to hold the borehole open. Water or drilling mud is typically not used. Cuttings are carried to the surface by the auger flights,



Figure 2-8

which allow visual classification of the soil. The advantage of the hollow stem auger is to permit the sampler and rod to be inserted down through the auger without removing the auger sections each time a sampler is inserted. The auger acts as a temporary casing. Samplers are inserted inside the auger casing to retrieve disturbed and undisturbed soil samples typically at 5 ft (1.5 m) intervals. Figure 2-7 demonstrates an auger drilling operation. Solid-stem augers are designated by the outside diameter of the auger flights. Common sizes are 3 inch, 4 inch, and 6 inch. Hollow-stem augers are designated by the inside diameter of the pipe. 3-1/4 inch and 4-1/4 inch are common sizes.

Solid-stem continuous flight augers consist of a solid steel central shaft with a continuous auger, typically available in 5 foot sections. The borehole is advanced by rotating the auger, which brings soil cuttings to the ground surface. Disturbed samples of soil may be taken from the augers, but in order to obtain undisturbed samples, the augers must be removed and a sampling tool placed in the bottom of the borehole. Continuous Flight Augers work well in stiff to very stiff fine-grained soils that maintain an open borehole, but do not work in very soft clays or sands and loose silts below the water table. These conditions require either wash boring or the use of Hollow Stem Augers (HSA).

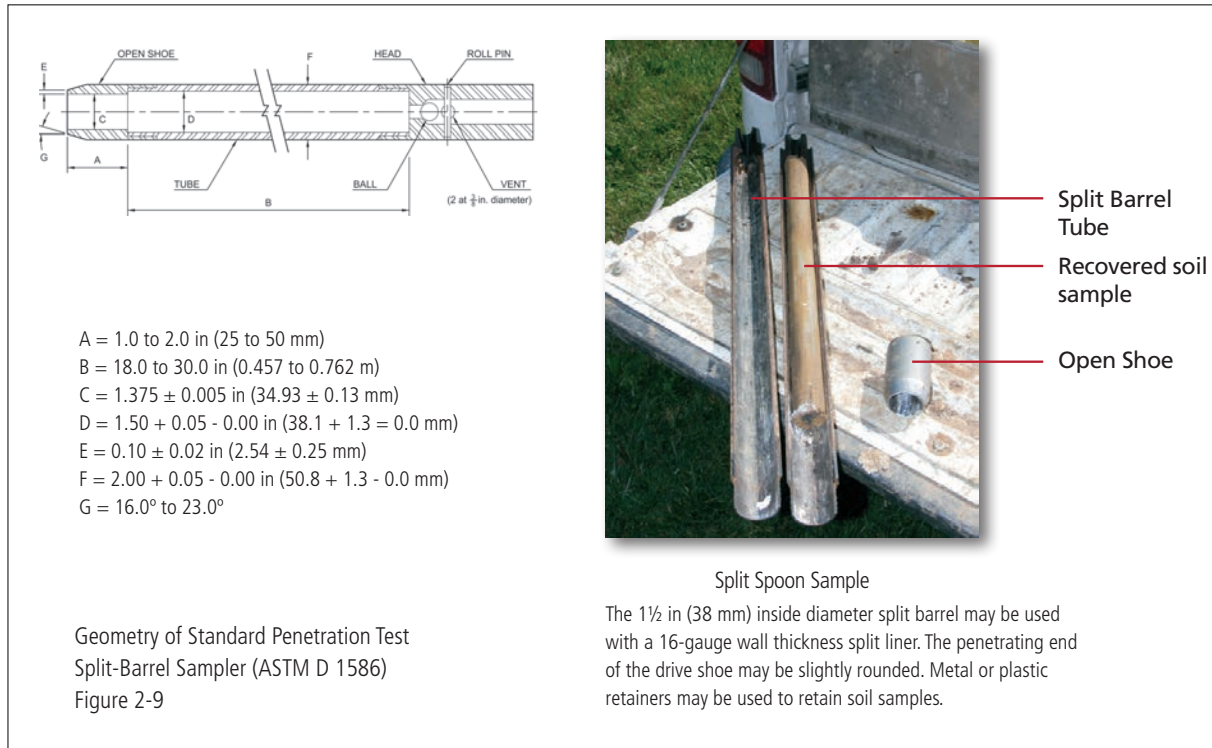
The groundwater table (GWT), or phreatic surface is defined as the elevation at which the pressure in the water is equal to that of the atmosphere. Information regarding the location of the groundwater table is very important to the design and construction of deep foundations – especially in granular soils. Careful observations should always be made and recorded, if circumstances permit, during exploratory drilling. It is customary to note the water level on completion of the hole and after allowing the hole to stand overnight or for 24 hours before backfilling. The use of drilling mud to stabilize the walls of the hole may preclude obtaining this information.

Soil Sampling

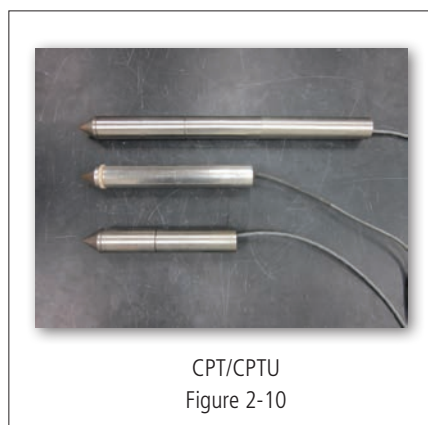
Geotechnical Site Investigations almost always include the collection of soil samples for identification and description, laboratory testing for soil classification and laboratory testing for soil strength and stiffness. There are two broad types of soil samples that are often collected; 1) disturbed samples, and 2) undisturbed samples. In general, disturbed samples may either be obtained from augers as previously discussed or more commonly they are obtained using the Standard Penetration Test (SPT). Undisturbed samples are typically obtained with thin-walled push tubes called Shelby Tubes (ST).

Standard Penetration Test and Sampling

The cuttings from exploratory drill holes are inadequate to furnish a satisfactory conception of the engineering characteristics of the soils encountered, or even the thickness and depths of the various strata. To obtain soil samples from test borings, a sampling spoon is attached to the drill rod and lowered to the bottom of the hole. The spoon is driven into the soil to obtain a sample and is then removed from the hole. The spoon is opened up and the recovery (soil sample length inside the spoon) is recorded. The soil is extracted from the spoon and inspected and described by the driller. A portion of the sample is placed in a glass jar and sealed for later visual inspection and laboratory determination of index properties.



The most common method of obtaining some information concerning relative density or the stiffness of in-situ soil consists of counting the number of blows of a drop weight required to drive the sampling spoon a specified distance into the ground. This dynamic sounding procedure is called the standard penetration test (SPT). The essential features include a drop hammer weighing 140 lb (63.5 kg) falling through a height of 30" (0.76 m) onto an anvil at the top of the drill rods, and a split spoon (SS) sampler having an external diameter of 2" (50.8 mm) and a length of 30" (0.76 m). The spoon is attached to the drill rods and lowered to the bottom of the drill hole. After the spoon reaches the bottom, the number of blows of the hammer is counted to achieve three successive penetrations of 6" (0.15 m). The number of blows for the first 6" is disregarded because of the disturbance that exists at the bottom of the drill hole. The number of blows for the second and third 6" increments are added and designated the standard penetration test (SPT), "N" value, or blow count. The data obtained from SPT tests are commonly recorded on soil boring logs relative to the sounding depth where the sample was taken. SPT values are widely used to correlate the shearing strength of soil for the design of shallow and deep foundations – including CHANCE® Helical Piles and ATLAS RESISTANCE® Piers. The SPT values also can assist in determining the depth of installation requirements for ATLAS RESISTANCE® Piers. Values of soil friction angle "φ" and cohesion "c" can be selected through correlation with the SPT "N" values. Details of the equipment and standardized procedures are



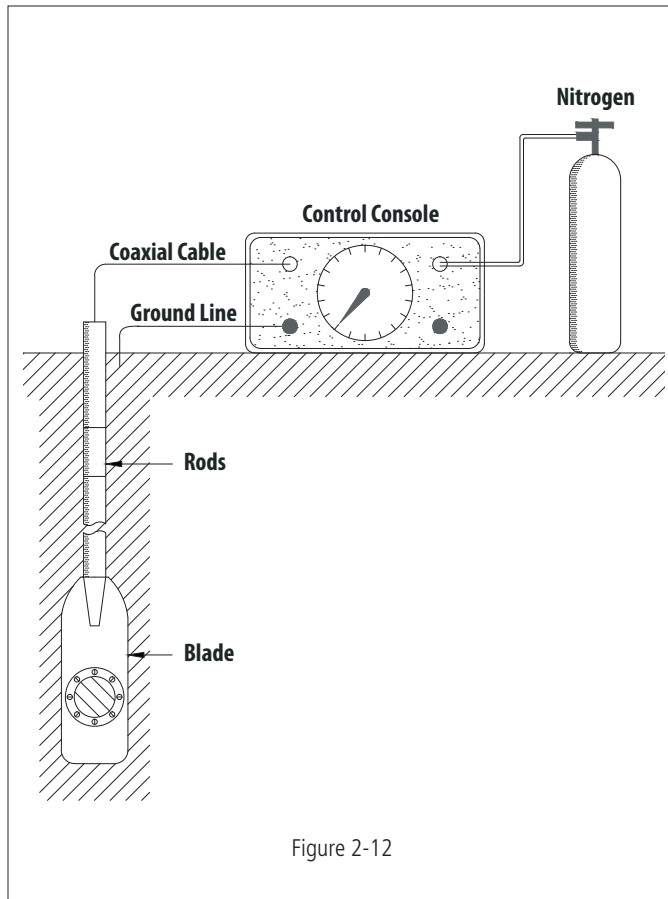


Figure 2-12

specified in ASTM D 1586. Figure 2-8 illustrates a drill crew conducting a Standard Penetration Test. The split spoon sampler is shown in Figure 2-9.

Undisturbed Samples

In general, soil samples taken from split spoon samplers are always considered disturbed to some degree for two reasons: 1) the sampler is driven into the soil, and 2) the split spoon is very thick. For soil samples to be used for laboratory analysis, the degree of disturbance of the samples must be reduced to a minimum. Reasonably satisfactory samples can be obtained in 50 and 76 mm samplers made of steel tubing about 1.5 mm thick. The lower ends are beveled to a cutting edge to give a slight inside clearance. This type of sampler is commonly referred to as a "Shelby tube". The Shelby tube is attached to the end of the drill rod and pushed vertically down into the soil to obtain an undisturbed sample. Hand samples or grab samples are sometimes taken from cuttings or test pits and are useful for soil classification and determining index properties. Details of the equipment and proper procedures for obtaining thin-walled Shelby Tube samples are specified in ASTM D1587.

IN-SITU TESTING METHODS

Cone Penetration Test (CPT) / Piezocone (CPTU)

The Cone Penetration Test consists of a cylindrical probe with a cone tip having an apex angle of 60° that is pushed slowly into the ground. The standard size cone has a diameter of 1.405 inch, which gives a projected end area of 10 cm^2 . Most cones also have a short section behind the tip that is called the sleeve. The force on the tip and the sleeve are measured independently during penetration to give the cone tip resistance, q_c , and the sleeve resistance, f_s . These values may then be used to evaluate changes in soil layering at a site and to estimate individual soil properties, such as shear strength and stress history. Some cones are also equipped with a porewater pressure sensor to measure the excess porewater pressure as the cone advances. This is called a piezocone. The cone tip resistance obtained from a piezocone is defined as q_t , the "effective" or corrected cone tip resistance since it is corrected for porewater pressure. A figure of a CPT and CPTU are shown in Figure 2-10.



Figure 2-13



Figure 2-11

Cone penetrometers cannot penetrate more than a few meters in dense sand, but they have been used to depths up to 60 m or more in soft soils. The friction ratio, defined as the friction resistance divided as the tip resistance can be correlated with the type of soil encountered by the penetrometer. Since no samples are obtained by use of cone penetrometers, borings and sampling are usually needed for definitive information about the type of soil being investigated.

Dilatometer Test (DMT)

The Dilatometer Test consists of a flat stainless steel blade with a circular, flexible membrane mounted on one side of the blade, as shown on Figure 2-11. The blade is pushed into the ground, much like a CPT or CPTU, but instead of providing continuous data, pushing is stopped every 1 foot. Immediately after pushing is stopped, the flexible membrane is expanded into the soil using nitrogen gas and a control console at the ground surface. Two pressure readings are taken; 1) the A-Reading, which is the pressure required to just initiate movement of the membrane into the soil, and 2) the B-Reading, which is the pressure required to expand the center of the membrane 1 mm into the soil. The two Readings are corrected for the stiffness of the membrane to give two pressure readings, P_0 and P_1 . P_0 and P_1 are then used along with the soil effective stress at each test depth to obtain estimates of specific soil properties such as shear strength, modulus, stress history and in-situ lateral stress. The specific requirements of the test are given in ASTM D6635.

Field Vane Test (FVT)

The Field Vane Test (FVT) or Vane Shear Test (VST) is used to measure the undrained shear strength and Sensitivity of medium stiff to very soft saturated fine-grained soils. It is considered one of the most reliable and direct in-situ test methods for determining undrained shear strength and the only in-situ test that may be used to determine Sensitivity. The test consists of inserting a thin four-bladed vane into the soil and rotating slowly to create a shear failure in the soil. The vane is usually rectangular with a height to diameter ratio (H/D) of 2, as shown in Figure 2-13. Initially, the maximum torque is measured to obtain the peak or undisturbed undrained shear strength. Then, the vane is rotated 10 times and the test is repeated to obtain the remolded undrained shear strength. The ratio of undisturbed to remolded strength is defined as Sensitivity, as previously described. The specific requirements of the test are given in ASTM D2573.

Mechanical Properties of Various Rocks, Table 2-5

| ROCK | YOUNG'S MODULUS AT ZERO LOAD (10^5 kg/cm ²) | BULK DENSITY (g/cm ³) | POROSITY (percent) | COMPRESSIVE STRENGTH (kg/cm ²) | TENSILE STRENGTH (kg/cm ²) |
|--------------|--|--------------------------------------|-----------------------|--|--|
| Granite | 2 - 6 | 2.6 - 2.7 | 0.5 - 1.5 | 1,000 - 2,500 | 70 - 250 |
| Microgranite | 3 - 8 | | | | |
| Syenite | 6 - 8 | | | | |
| Diorite | 7 - 10 | | | 1,800 - 3,000 | 150 - 300 |
| Dolerite | 8 - 11 | 3.0 - 3.05 | 0.1 - 0.5 | 2,000 - 3,500 | 150 - 350 |
| Gabbro | 7 - 11 | 3.0 - 3.1 | 0.1 - 0.2 | 1,000 - 3,000 | 150 - 300 |
| Basalt | 6 - 10 | 2.8 - 2.9 | 0.1 - 1.0 | 1,500 - 3,000 | 100 - 300 |
| Sandstone | 0.5 - 8 | 2.0 - 2.6 | 5 - 25 | 200 - 1,700 | 40 - 250 |
| Shale | 1 - 3.5 | 2.0 - 2.4 | 10 - 30 | 100 - 1,000 | 20 - 100 |
| Mudstone | 2 - 5 | | | | |
| Limestone | 1 - 8 | 2.2 - 2.6 | 5 - 20 | 300 - 3,500 | 50 - 250 |
| Dolomite | 4 - 8.4 | 2.5 - 2.6 | 1 - 5 | 800 - 2,500 | 150 - 250 |
| Coal | 1 - 2 | | | 50 - 500 | 20 - 50 |
| Quartzite | | 2.65 | 0.1 - .05 | 1,500 - 3,000 | 100 - 300 |
| Gneiss | | 2.9 - 3.0 | 0.5 - 1.5 | 500 - 2,000 | 50 - 200 |
| Marble | | 2.6 - 2.7 | 0.5 - 2 | 1,000 - 2,500 | 70 - 200 |
| Slate | | 2.6 - 2.7 | 0.1 - 0.5 | 1,000 - 2,000 | 70 - 200 |

Notes:
1) For the igneous rocks listed above, Poisson's ratio is approximately 0.25
2) For a certain rock type, the strength normally increases with an increase in density and increase in Young's Modulus (after Farmer, 1968)
3) Taken from Foundation Engineering Handbook, Winterkorn and Fong, Van Nostrand Reinhold, page 72

The maximum torque (T) is measured during rotation and for a vane with $H/D = 2$ the undrained shear strength is determined from:

$$s_u = (0.273T)/D^3 \quad \text{Equation 2- 5}$$

Vanes are available in different sizes to suit the soil at a particular site. The Field Vane Test may be especially useful in evaluating sites for helical piles/anchors as it may give some insight to the engineer into the degree of disturbance and strength reduction that the soil may experience during installation, depending on the Sensitivity. It is important that the exact geometry of the vane (e.g., H, D, thickness of blades) and test procedures used be described in a Geotechnical Report so that the engineer may make any adjustments to the test results for the equipment used.

Helical Probe

Shear strength also can be estimated by installing a helical pile “probe” and logging installation torque vs. depth. The torque values can be used to infer shear strength based on the torque-to-capacity relationship discussed in Section 6.

Rock Coring and Quality of Rock Measurement

When bedrock is encountered, and rock anchors are a design consideration, a continuous rock core must be recovered to the depth or length specified. Typical rock anchors may be seated 20 ft. or 30 ft. into the rock formation.

In addition to conducting compressive tests on the recovered rock core samples (See Table 2-5), the rock core is examined and measured to determine the rock competency (soundness or quality). The rock quality designation (RQD) is the most commonly used measure of rock quality and is defined as:

$$RQD = \frac{\sum \text{Length of intact pieces of core (>100 mm)}}{\text{Length of core run}}$$

The values of RQD range between 0 and 1.0 where an RQD of 0.90 or higher is considered excellent quality rock.

Helical piles/anchors rotated or torqued into the ground cannot be installed into hard, competent bedrock. However, in upper bedrock surfaces comprised of weathered bedrock material such as weathered shale or sandstone, the helix plates can often be advanced if the RQD is 0.30 or less.

The presence of an intact bedrock surface represents the ideal ground condition for ATLAS RESISTANCE® Piers. In this ground condition, the ATLAS RESISTANCE® Pier is installed to the rigid bearing surface represented by the bedrock layer.

Laboratory Testing of Recovered Soil Samples

Laboratory testing is typically part of a subsurface investigation and may vary in scope depending upon project requirements or variability in soil conditions. Some of the more typical laboratory tests are described below:

Classification / Characterization Tests

- **Visual Classification** – Samples collected during the drilling operations should be visually classified. Every recovered sample from the field boring and sampling program is inspected visually and given a visual description as to its collection depth, percent recovery, moisture conditions, soil color, inclusion type and quantity, approximate strength, odor and composition (See Table 2-4). In addition to this visual classification, a representative number of samples are selected to conduct the following tests:
- **Water Content** – measures the amount of moisture in the soil. Moisture or water content is measured by weighing a soil sample taken from the field on a laboratory scale. The soil sample is then placed in a standard oven for a sufficient time to allow all the moisture to evaporate. After being removed from the oven, the soil sample is weighed again. The dried weight is subtracted from the original weight to determine the water weight of the sample. These methods are also used to determine the total (wet) unit weight and the dry unit weight.

Sample Boring Log in Coarse-Grained Soil, Table 2-6

| Project No.: 12-1122 | | | | Boring Log | | | | Rig: CME 75 with 140 lb Auto Hammer | | | | | | | |
|--------------------------------------|--------|--|------------|---------------------|--------------|--------|------|-------------------------------------|------------|--------|--|-------------------------------------|----|-----------------|--|
| Project: Doe Run Test Borings - 2012 | | | | | | | | Location: Leadwood, MO | | | | | | | |
| Client: Hubbel Power Systems | | | | | | | | Driller: MAS | | | | | | | |
| Boring No.: 1 | | | | | | | | | | | | | | | |
| SUBSURFACE PROFILE | | | | | | | | | | SAMPLE | | Standard Penetration Test blows/ft. | | Water Content % | |
| Depth (ft.) | Symbol | Description | Qp, T.S.F. | Dry Density, P.C.F. | Depth/Elev. | Number | Type | Blows/ft. | Qu, T.S.F. | | | Wp | WI | | |
| 0 | | Ground Surface | | | 100.0 | | | | | | | | | | |
| | | Crushed Stone: Poorly Graded Sand with Silt (SP-SM), Light Gray, Trace Gravel, Fine to Coarse, Dry | | | | 0 | HA | | | | | | | | |
| | | (SP-SM), Trace Gravel, Fine to Coarse, Medium Dense, Dry | | | | 1 | SS | 29 | | | | | | | |
| | | Blow Sequence = 6-13-16 | | | | | | | | | | | | | |
| | | Recovery = 14" | | | | | | | | | | | | | |
| 5 | | (SP-SM), Trace Gravel, Fine to Coarse, Medium Dense, Dry | | | | 2 | SS | 27 | | | | | | | |
| | | Blow Sequence = 10-13-14 | | | | | | | | | | | | | |
| | | Recovery = 18" | | | | | | | | | | | | | |
| | | (SP-SM), Fine to Medium, Medium Dense, Moist | | | | 3 | SS | 15 | | | | | | | |
| | | Blow Sequence = 8-8-7 | | | | | | | | | | | | | |
| | | Recovery = 16" | | | | | | | | | | | | | |
| 10 | | (SP-SM), Fine to Medium, Medium Dense, Moist | | | | 4 | SS | 10 | | | | | | | |
| | | Blow Sequence = 3-5-5 | | | | | | | | | | | | | |
| | | Recovery = 19" | | | | | | | | | | | | | |
| 15 | | (SP-SM), Fine to Medium, Loose, Moist | | | | 5 | SS | 8 | | | | | | | |
| | | Blow Sequence = 2-4-4 | | | | | | | | | | | | | |
| | | Recovery = 17" | | | | | | | | | | | | | |
| 20 | | (SP-SM), Fine to Medium, Medium Dense, Moist | | | | 6 | SS | 12 | | | | | | | |
| | | Blow Sequence = 3-6-6 | | | | | | | | | | | | | |
| | | Recovery = 18" | | | | | | | | | | | | | |
| 25 | | (SP-SM), Fine to Medium, Loose, Moist | | | | 7 | SS | 7 | | | | | | | |
| | | Blow Sequence = 2-3-4 | | | | | | | | | | | | | |
| | | Recovery = 15" | | | | | | | | | | | | | |
| 30 | | (SP-SM), Fine to Medium, Loose, Moist/Wet | | | | 8 | SS | 5 | | | | | | | |
| | | Blow Sequence = 1-2-3 | | | | | | | | | | | | | |
| | | Recovery = 15" | | | | | | | | | | | | | |
| | | End of Boring @ 31½ Ft. | | | 68.5 31.5 | | | | | | | | | | |

Drill Method: 3 1/4" HSA with AW Rod

Boring Started: 9-10-2012

Boring Completed: 9-10-2012

Tested By: N/A

Logging By: PEB

GEOTECHNICS

Soil & Material Testing

Groundwater Elev. During Drilling: ▽ 69.0

Groundwater Elev. @ Comp.: ▽

Groundwater Elev. @ 1 Hrs.: ▽

Boring Location: West Boring

Sheet 1 of 1

- **Particle Size Analysis** – measures the distribution of particle sizes within the soil sample.
- **Atterberg Limits** – Liquid Limit (LL), Plastic Limit (PL), Shrinkage Limit (SL), and Plastic Index (PI) – applies to cohesive types of soil and is a measure of the relative stiffness of the soil and potential for expansion. Index properties (LL, PL, SL, and PI) are determined using specially developed apparatus and procedures for performing these tests. The equipment, specifications and procedures are closely followed in ASTM D 4318 Classification / Characterization Tests. The Liquid Limit and the Plastic Limit are particularly important since they may be used along with the natural water content to determine the Liquidity Index.

STRENGTH CHARACTERISTICS

In some instances undisturbed soil samples are recovered in the field using a thin wall Shelby tube. These recovered samples are tested either in triaxial or direct shear tests to determine directly the friction angle " ϕ " and the cohesion " c " of the soil. For cohesive (clay) soil samples, an unconfined compression test "UC" is often conducted. The unconfined compression test is used to determine the unconfined compression strength " q_u " of the clay soil. The cohesion of the clay sample is then taken to be one-half of " q_u ". The unconfined compression test is commonly performed due to its low cost; however the results tend to be conservative and simulate only total stress conditions with no confining pressure which may not be appropriate for the project. For granular soils, the Direct Shear test is a relatively inexpensive test to determine the soil friction angle and may also be used for undrained testing of cohesive samples. More refined laboratory testing may be appropriate for large projects and may offer a cost saving potential by justifying higher soil strength than using less sophisticated test methods. Some of the more complex strength tests include, Consolidated Drained (CD), Consolidated Undrained (CU) and Unconsolidated Undrained (UU) Triaxial tests for total and effective stress paths at project specific confining stresses.

THE GEOTECHNICAL REPORT

The geotechnical report provides a summary of the findings of the subsurface investigation, and the results of the laboratory testing. Geotechnical reports usually include an introduction detailing the scope of work performed, site history including geology, subsurface conditions, soil profile, groundwater location, potential design constraints such as seismic parameters and corrosion potential, foundation options, allowable load capacities, and an appendix which includes soil boring logs. Soil boring logs provide a wealth of information that is useful in the design of CHANCE® Helical Piles and ATLAS RESISTANCE® Piers. Boring logs come in variety of designs since there is no standard form, but they contain basically the same type of information – most of which has been discussed in this section. Items to expect on a soil boring are: total boring depth, soil profile, description of soil samples, sample number and type, Standard Penetration Test N-values, moisture content, Atterberg limits, unconfined compression strength or undrained shear strength (cohesion), groundwater table location, type of drilling used, type of SPT hammer used, and sample recovery. An example boring log is shown in Table 2-6 & 2-7. Table 2-6 is a soil boring taken in a coarse-grained sand soil. Table 2-7 is a soil boring taken in a fine-grained clay soil.

Sample Boring Log in Fine-grained Soil, Table 2-7

| Project No.: 09-1219 | | | | | | | | | | Rig: CME 55 | | | |
|---|--------|---|------------|---------------------|-------------|--------|------|-----------|------------|---------------------------------------|----|-----------------|--|
| Project: Mexico and Eaton Dam Drill Sites | | | | | | | | | | Location: Mexico/Park Hills, Missouri | | | |
| Client: Chance Civil Construction | | | | | | | | | | Driller: MAS | | | |
| Boring No.: Mexico | | | | | | | | | | | | | |
| SUBSURFACE PROFILE | | | | | SAMPLE | | | | | Standard Penetration Test blows/ft. | | Water Content % | |
| Depth (ft.) | Symbol | Description | Qp, t.s.f. | Dry Density, P.C.F. | Depth/Elev. | Number | Type | Blows/ft. | Qu, T.S.F. | | Wp | WI | |
| 0 | | Ground Surface | | | 0.0 | | | | | | | | |
| | | Fill: Yellow Brown Silty Clay, w/Sand, (CL) | | | -2.5 | 0 | HA | | | | | | |
| | | Fill: Yellow Brown Micaceous Clay, Trace Sand, Stiff, (CH) | 3.50 | | 2.5 | 1 | SS | 10 | | | | | |
| 5 | | Fill: Mottled Reddish Brown, Trace Sand and Gravel, Very Stiff, (CH) | 3.50 | | -7.5 | 2 | SS | 18 | | | | | |
| | | Fill: Light Gray mottled Yellow Brown Clay, w/Sand and Gravel, Pieces of Coal and Shale, Very Stiff, (CH) | 4.00 | | 7.5 | 3 | SS | 16 | | | | | |
| 10 | | Fill: W/Pieces of Limestone, Stiff, (CH) | | | -12.5 | 4 | SS | 8 | | | | | |
| | | Light Gray Weathered Micaceous Clay (Shaly Residuum), Hard, (CH) | 4.5+ | | 12.5 | 5 | SS | 57 | | | | | |
| 15 | | Hard, (CH) | | | | 6 | SS | 53 | | | | | |
| | | Hard, (CH) | | | | 7 | SS | 105/10 | | | | | |
| 20 | | Hard, (CH) | | | -21.5 | 8 | SS | 50/4" | | | | | |
| | | End of Boring @ 21 1/2 Ft. | | | 21.5 | | | | | | | | |
| 25 | | | | | | | | | | | | | |
| 30 | | | | | | | | | | | | | |

| | | |
|------------------------------|---|--------------------------------------|
| Drill Method: 3 1/4" HSA | GEOTECHNICS Soil & Material Testing | Groundwater Elev. During Drilling: ▽ |
| Boring Started: 11-23-2009 | | Groundwater Elev. @ Comp.: ▽ |
| Boring Completed: 11-23-2009 | | Groundwater Elev. @ Hrs.: ▽ |
| Tested By: | | Boring Location: Client Provided |
| Logging By: PEB/BJS | | Sheet 1 of 1 |

Problem Soil Conditions

All natural materials, such as soil, will exhibit conditions of variability that may make a single solution inadequate for inevitable problems that arise. It is wise to remember Dr. Terzaghi's emphasis to have a secondary solution ready when dealing with the variability of soils.

Deep Fill, Organic and Collapsible Soils

The existence of deep fills, organic and collapsible soils on a given project site are typically known before the start of the project. This is usually determined during the subsurface investigation by means of drilling or sounding. However, on large projects like an underground pipeline or transmission line that covers many miles, these soils may occur in undetected pockets and hence present a potential problem. The best solution is to be aware of the possibility of their existence and be prepared to install CHANCE® Helical Piles and ATLAS RESISTANCE® Piers deeper to penetrate through this material into better bearing soil. It is not recommended to locate the helical bearing plates or the tip of the ATLAS RESISTANCE® Pier in these soils.

Loose Liquefiable Soils

Some deposits of saturated sand and silty sand are naturally loose and may be prone to lose strength or liquefy during an earthquake or other dynamic loading. These soils are typically identified by very low SPT N-values (typically less than about 6) and should be viewed with caution.

Sensitive Clays

Some marine clay deposits are also very sensitive and can lose most of their shear strength when disturbed and when loaded dynamically. These deposits are typically identified with Liquidity Index greater than about 1.2.

Expansive Soils

Expansive soils exist all over the earth's surface, in nearly every region. These soils are often described as having high shrink-swell behavior since they can also shrink if dried out. The natural in-place weathering of rock produces sand, then silt, and finally clay particles – hence the fact that clay is a common soil type. Most clay soils exhibit volume change potential depending on moisture content, mineralogy, and soil structure. The upward forces (swell pressure) of expansive clay may far exceed the adfreeze forces generated by seasonally frozen ground, yet foundations continue to be founded routinely in expansive soil with no allowance for the potential expansion. Foundations should be designed to penetrate below the expansive soil's active zone, or be designed to withstand the forces applied the foundation, e.g., to prevent "slab dishing" or "doming." The active zone is defined as the depth of expansive soil that is affected by seasonal moisture variation. Another method used to design foundations on expansive soil is to prevent the soil's moisture content from changing. Theoretically, if the moisture content does not change, the volume of the clay soil will not change. This is typically difficult to control.

The tensile strength of deep foundations must be sufficient to resist the high tensile forces applied to the foundation by expansive soil via skin friction within the active zone. As an expansive soil swells or heaves, the adhesion force between the soil and the side of the foundation can be of sufficient magnitude to "jack" a foundation out of the ground. CHANCE® Helical Piles are a good choice in expansive soils due to their relatively small shaft size – which results in less surface area subjected to swell pressures and jacking forces. Isolating footings, slabs, and grade beams from subgrade soils by using void form is a typical detail used in areas like Denver, Colorado, where expansive soil is present. The void form isolates the structure from contact with the expansive soil, thereby eliminating the destructive effects of swell pressures.

A Plasticity Index (PI) greater than 25 to 30 is a red flag to the geotechnical engineer. A $PI \geq 25$ to 30 indicates the soil has significant volume change potential and should be investigated further. There are fairly simple tests (Atterberg, soil suction test, swell potential) that can be conducted but should be practiced by the informed designer.

Seasonally Frozen Ground

The most obvious soil in this category is the frost susceptible soils (typically, silt) as illustrated by the growth of frost needles and ice lenses in freezing weather. This leads to a commonly observed expansion phenomenon known as frost heave. Frost heave is typically observed on roadbeds, under concrete slabs, and along freshly exposed cuts. Capillary breaks and vapor barriers in conjunction with proper drainage will do much to control this problem, before CHANCE® Helical Piles or ATLAS RESISTANCE® Piers are installed.

A subcategory of this condition is seasonal permafrost. If possible, these ice lenses should be penetrated and not relied on for end bearing.

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